Technical Report REMR-GT-20
October 1993

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Station

Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Evaluation of Overturning Analysis for Concrete Structures on Rock Foundations

by *Shannon and Wilson, Inc.*
Engineering and Applied Geosciences

WES

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Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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by Shannon and Wilson, Inc.
Engineering and Applied Geosciences
St. Louis, MO 63141-7126

Final report

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Monitored by Geotechnical and Structures Laboratories
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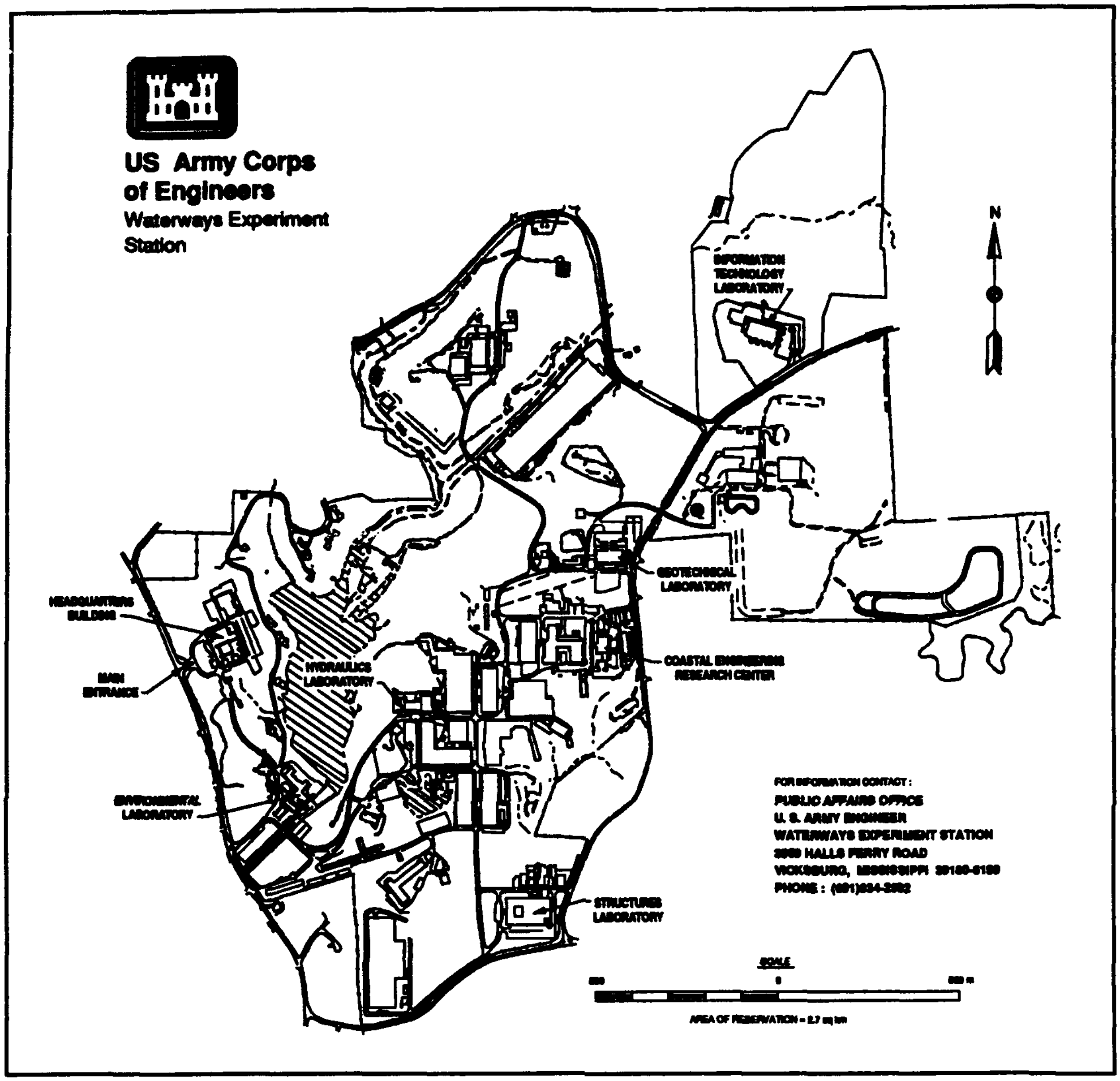

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Contents

Preface

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Geotechnical Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research program. The work was performed under Civil Works Research Work Unit 32648, "Geomechanical Modeling for Stability of Existing Gravity Structures." The REMR Technical Monitor was Mr. Wayne Swartz (CECW-EG).

Mr. William N. Rushing (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HWUSACE. Mr. James E. Crews (CECW-O) and Dr. Tony C. Liu (CECW-EG) served as the REMR overview Committee. The REMR Program Manager was Mr. William F. McCleese, U.S. Army Engineer Waterways Experiment Station (WES). Mr. Jerry S. Huie, Geotechnical Laboratory (GL), WES, was the Problem Area Leader.

The study was performed by Shannon and Wilson, Inc., under Contract No. DACW39-86-M-4062 to WES. Mr. Robert D. Bennett was Principal Investigator. This work was conducted under the direct supervision of Mr. Huie and under the general supervision of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL. Dr. Paul F. Hadala was Assistant Director, GL, and Dr. William F. Marcuson III was Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement can be converted to SI units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
inches	2.54	centimeters
pounds (force) per square feet	47.88026	pacals
pounds (force) per square inches	6.894757	kilopascals
tons (force)	8.896444	kilonewtons

1 Introduction

Recent interest has developed in the review of the stability of concrete gravity structures on rock foundations. This interest has been stimulated by the large number of such structures and the growing recognition of the need to realistically determine if marginal stability exists that requires remedial action or repair work. It is important to have simple analytical methods available to screen the various gravity structures for stability. If the simple screening methods indicate a concern with the stability of the structure, the cost of more sophisticated and accurate methods of analysis such as finite element methods may be justified.

Purpose

The purpose of this study is to review the method presently used by the U.S. Army Corps of Engineers (CE) to analyze overturning.

Scope

This report includes a discussion of the present method of overturning analysis and a discussion of the underlying assumptions; the contact stresses under the base of the structure; and the effects of strain compatibility, compaction-induced lateral stresses, and wall friction on the driving and resisting moments. Recommendations are made for modifications to the present method of analysis, and areas for future studies where additional improvements could be made are discussed.

The present scope of work addresses primarily the mechanical aspects of the method of stability analysis. The site geologic conditions will have a significant effect on the assumptions made in the analysis and the shear strength parameters used; however, a discussion of these issues is beyond the present scope of work.

2 Review of Existing Overturning Analysis

The overturning analysis presently used by the CE involves satisfying static equilibrium conditions for vertical forces and moments. By making an assumption regarding the uplift pressure, summing the vertical forces, and summing the moments about the base, the magnitude and location of the foundation reaction can be found. The base pressure distribution is then calculated, and the assumption regarding uplift pressure is checked. If the base pressure distribution indicates that a "noncompression" zone is present at the heel of the structure, the uplift pressure diagram is modified to include full hydrostatic pressure along the "noncompression" zone. The analysis is then iterated until the assumed and calculated locations of the "noncompression" zone agree. The present criteria for overturning stability are based on the location of the foundation reaction and a linear base pressure distribution. For the normal operating condition, the foundation reaction must generally fall within the middle third to middle half of the base depending on loading condition, resulting in 100 to 75 percent of the base in compression. While a factor of safety is not explicitly calculated, the requirement for the foundation reaction to be within the middle third to middle half of the structure's base results in an implicit factor of safety.

The reasons for the present criteria are probably three:

a. An implicit factor of safety is provided.

b. No or little "tension" is permitted on the back edge of the structure that might permit excessive progressive uplift pressures to develop on the base of the structure.

c. No uncertainties are developed in the shear strength along the base of the structure due to asperities as a result of some unknown portion of the base no longer being in compression.

3 Base Pressure Distribution

The state of stress in the foundation immediately below a concrete gravity structure is controlled by two major components: (a) the foundation reaction resulting from lateral earth and water pressures, the weight of the structure, and the weight of the backfill; and (b) uplift pressures that are a function of the water level on both sides of the wall and geologic discontinuities within the rock mass. Each of these components is discussed in turn below.

Foundation Reaction

The present method of calculating the component of the base pressure resulting from the foundation reaction makes the following assumptions: (a) the foundation is flexible but cut away vertically at the extent of the structure, and the dam is infinitely stiff (an excellent conceptional discussion of the base pressure distribution relative to the stiffness of the foundation and structure is presented in Hinds, Creager, and Justin (1957)); (b) the state of stress at the concrete-rock contact and on the critical failure surface are the same; that is, the state of stress does not change with depth; (c) the presence of the backfill behind the wall has no effect on the state of stress under the wall; and (d) the structure-foundation interface cannot support any tensile stresses. The first three assumptions are rarely, if ever, true for gravity structures on rock foundations, and the last assumption may be too severe for some cases.

Relative Stiffness

Plate 1 shows the contact stress, p, divided by the applied load, q, below an infinite strip subjected to a uniform pressure for different relative stiffnesses of the structure with respect to the foundation (Borowicha 1936). It can be seen that for relative stiffnesses greater than $\pi/3$, or approximately 1, the strip can be considered to be perfectly rigid with respect to the foundation with minor error.

For a concrete modulus of 3,000,000 psi[1] and a foundation modulus of 13,000,000 psi, the relative stiffness, K, is approximately 1, which means that the structure would be rigid relative to the foundation unless the foundation modulus exceeds 13,000,000 psi. This indicates that the structure would still be considered rigid with respect to the foundation even though the foundation modulus is quite high.

Therefore, formulae for contact stresses below rigid structures would more realistically represent the state of stress below typical concrete gravity structures than the CE formulae presently being used. Along with a plot of the contact stresses for the structure shown in Plate 2 calculated using both the present CE analysis and the rigid structure-flexible base formulae, the rigid structure-flexible base formulae are shown in Plate 3.

It can be seen that the "noncompression" zone that develops along the back edge of the structure using the CE analysis does not appear for this particular geometry when the rigid structure-flexible base formulae are used because of the high edge stresses predicted by rigid structure-flexible base theory. This may explain why pore pressure measurements made along the back edge of structures, which according to the CE analysis have developed a "noncompressive" zone, do not always show full hydrostatic pressures as would be expected. In fact, the back edge of the structure may be fully in compression.

The high edge pressures shown in Plate 3 may not fully develop because of limitations imposed by the shear strength of the foundation material. Plastic deformation will occur when the contact stresses exceed the shear strength of the foundation material, thus creating an upper limit for the stresses that can be applied at a given point. Stresses in excess of those required to cause plastic deformation will be redistributed, in this case towards the center of the structure, thus flattening the shape of the stress distribution curve. This effect will be more pronounced for weaker foundation materials. However, for structures on rock, field measurements (Sisko and Johnson 1964) have shown that the curvature of the contact stress distribution is still significant.

The assessment of the effects of plastic failure requires additional considerations regardiing the selection of the shear strength parameters of the foundation material. For structures on rock foundations, the shear strength along discontinuities or planes of weakness may be the controlling factor in sliding analyses. The determination of these shear strength parameters has been discussed in detail by Nicholson (1983). However, the failure mechanism induced by overturning is completely different from that induced by sliding. The shear strength along a different set of

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

discontinuities or even through intact rock is likely to control the development of plastic failure due to overturning.

Depth Effects

The distribution of contact stresses as a function of depth may also mitigate the effects of the high contact stresses along vertical depth profiles through the edges of the structure. Simple elastic solutions that describe the distribution of stress with depth for rigid bodies are not readily available. However, it is likely that the high stresses dissipate rapidly with depth, and at some depth the stress distribution may approach that presently used by the CE. The effect of this stress dissipation with depth will depend primarily on the location of the failure surface with depth.

Plate 4 illustrates the base pressure distribution with depth for the structural configuration shown in Plate 2. The contact stress distribution was calculated using the CE method rather than the rigid body formulae, thus resulting in a tension zone at the heel of the structure. For the sake of simplicity, it is assumed that tension (shown as negative stress) can be carried by the rock mass, and no increase in uplift pressure in the tension zone is included. The stress distributions at depth were calculated using formulae for uniform applied stresses (i.e., assuming that the structure is flexible compared to the foundation) because solutions for rigid bodies are not available.

The tensile stress at the heel was computed to be -720 psf. It can be seen that, at a depth of 3 in., the tensile stress has decreased to approximately -400 psf, and within a depth of 3 ft, the tensile stress has disappeared altogether.

Backfill Effects

The present analysis assumes that the weight of the backfill behind the wall has no effect on the state of stress under the wall. However, the backfill represents a significant compressive stress in the immediate area where tensile stresses are likely to develop. Plate 4 also illustrates the effects of the backfill compressive stress with depth. When the effects of the backfill are considered, the tensile stresses generated at the heel of the structure are dissipated within a depth of less than 1 ft.

Tensile Stresses

While the above discussions indicate that the actual tensile stresses under a gravity structure may be nonexistent, or considerably less than predicted by the present CE analysis, it is also true that some rock masses can carry tension. For heavily jointed rock masses, the assumption of zero tensile strength is justified. However, for rock masses with moderate to wide joint spacings, particularly if the joint surfaces are clean and unsheared, a small but significant tensile strength may be present.

Uplift Pressures

Uplift pressures are controlled by the water level on both sides of the wall along with the drainage conditions, grout curtain, and base pressure distribution under the wall. As long as 100 percent of the base is in compression, uplift pressures are assumed to vary linearly between the water levels on opposite sides of the wall, but may be reduced if drainage under the wall is provided.

One of the most stringent requirements of the present CE analysis is that full hydrostatic pressure be applied to the base of the structure wherever a "noncompression" zone is indicated in the foundation reaction. The assumption implicit in this requirement is that a crack, hydraulically connected to the pool behind the wall, opens up when the rock mass at the heel of the structure is not being compressed.

As discussed previously, the noncompression zone is likely to be considerably smaller than predicted by the present analysis and may not exist at all at the depth of the critical failure surface. A small tensile strength in the rock mass would further reduce the likelihood of a crack appearing at the heel of the structure. Thus, the application of full uplift pressure along the hypothetical tension crack is probably overly conservative.

4 Strain Compatibility

One of the fundamental principles of earth pressure theory is that the magnitude of the lateral earth pressure against a structure is a function of the deformation of the structure. It is thus imperative, that the coefficients of lateral earth pressure used in the stability analysis be consistent with the known deformation conditions of the structure. Current practice (U.S. Corps of Engineers 1961) is to assume that deformations of a concrete gravity structure on a rock foundation (except rock with low elastic modulus) will not be sufficient to mobilize active or passive earth pressures on the wall. Thus, at-rest earth pressures are used on both sides of the wall if the wall is not on rock with a low elastic modulus.

Deformation Conditions

The elastic deformation of the foundation under the loads applied by a rigid body can be easily calculated. The elastic settlement of the structure is not of interest since vertical movement will do little to mobilize the shear strength of the backfill and decrease the earth pressure on the structure. However, the elastic rotation of the structure will have the tendency to mobilize the shear strength of the backfill and cause a reduction of lateral earth pressure from the at-rest state towards the active state. The elastic rotation of the structure is given by:

$$\alpha = \frac{4M(1 - \nu^{2})}{\pi E b^2} \qquad (1)$$

where

α = Angle of rotation in radians

M = Applied moment in units of moment per unit length, e.g., ft-lb/ft

ν = Poisson's ratio

E = Young's modulus

b = Halt width of base of structure

For the typical configuration shown in Plate 2, this results in a rotation of 3×10^{-4} radians. Studies (Sherif, Fang, and Sherif 1984) have shown that the active earth pressure is fully mobilized for rotational deformations on the order of 5×10^{-4} to 1×10^{-3} radians, depending on the modulus of the backfill material (construction sequences can induce higher residual pressures against a wall). Thus, for the configuration in Plate 2, which includes a relatively low rock modulus, earth pressures lower than at-rest would be expected on the back of the wall. For higher values of the rock modulus, deformations would be less and might not be sufficient to reduce earth pressures significantly below at-rest values.

The deformations required to mobilize passive earth pressures are considerably greater than those required to mobilize active earth pressures. The elastic rotation of the structure is likely to be too low to mobilize the full passive earth pressures.

Compaction-Induced Stresses

For cases where elastic rotations of the wall are small and at-rest conditions apply on both sides of the wall, compaction-induced stresses in the backfill should be considered.

General models of the development of lateral stresses in a soil mass have been developed by a number of researchers (Duncan and Seed 1986, Ingold 1979). The general features of the various models are quite similar and are illustrated in Plate 5. In general, the state of stress in the soil mass during virgin loading (i.e., loading in excess of that previously experienced by the soil) will plot on the *Ko* line for uniform loads such as those applied by increasing thicknesses of overburden. Upon unloading, the horizontal stresses relax at a lower rate than the vertical stresses, thus resulting in a higher ratio of horizontal to vertical stress than existed prior to unloading. This stress difference is limited by the passive failure condition. Thus, if the unloading curve reaches the *Kp* line, the unloading curve then follows the *Kp* line.

Lateral stresses generated by loads of finite lateral extent, such as those applied by compaction equipment, can be considerably higher than those generated by uniform loads of the same magnitude. Plate 6 shows the horizontal stress distribution with depth, as calculated by elastic theory, for a 10-ton vibratory roller acting at a distance of 2 ft away from the wall. The limiting conditions of passive and at-rest earth pressures are also shown as the *Kp* and *Ko* lines, respectively, using the soil parameters shown in Plate 2. The roller is simulated by a line load acting over the width of the roller, assumed to be 6 ft, with the effective dynamic weight

of the roller taken to be twice the static weight of the roller. The solution for an elastic half space is multiplied by a factor of 2 to account for the presence of the rigid wall in close proximity to the applied load, as recommended by Duncan and Seed (1986). It can be seen that the lateral stresses calculated directly by elastic theory are considerably larger than those calculated by multiplying the corresponding vertical stress by *Ko*.

The lateral stress at a point in the soil backfill as a result of both compaction stresses and increasing overburden stress can be explained as follows. When the compaction equipment passes over the point in the soil mass, high lateral stresses are created. For large rollers of the type likely to be used in lock and dam construction, this lateral pressure may be in excess of the passive pressure (as seen in Plate 6), and shearing of the soil occurs, limiting the lateral pressure to the passive condition. When the compaction load is removed, the vertical stresses return to their original value, defined by the weight of the overburden, while the horizontal stresses retain some portion of their maximum previous value.

As additional lifts of soil are placed above the point in question, the incremental vertical stress increase caused by the compaction equipment is diminished, as shown in Plate 6. At the same time, the overburden pressure continues to grow, and at some critical depth the lateral stresses related to the overburden pressure exceed the maximum previous lateral stress experienced by the soil. At this point, the lateral stress in the soil is simply related to *Ko* and the overburden pressure.

The resultant lateral stress distribution on a rigid structure is shown by the envelope drawn in Plate 6. This envelope is controlled by the *Ko* line, the *Kp* line, and the intersection between the elastic horizontal stress curve and the *Kp* line. At depth, the overburden stresses overshadow any compaction induced stresses, and the lateral stress disttibution follows the *Ko* line. At some critical shallower depth, the compaction stresses become significant, and the lateral stress becomes higher than that defined by *Ko*. The magnitude of the lateral stress is then defined by the maximum past lateral stress experienced by the soil as a result of compaction. This maximum pressure may be limited by the passive failure condition, as is the case of the example shown in the Plate 6. Close to the ground surface, the compaction-induced lateral stresses reach the limiting passive failure condition and follow the *Kp* line.

The principal difference between the various models is in the slope of the unloading curve shown in Plate 5. If the curve is assumed to be horizontal, all of the horizontal stresses generated by the compaction plant are retained by the soil, except as limited by-the passive failure condition. This is the case shown by the envelope in Plate 6. If the curve is assumed to have some slope, then only a portion of the horizontal stresses related to compaction are retained. This portion ranges from 40 to 100 percent of the maximum previous value, depending on the model.

The model proposed by Ingold (1979) reduces the complex interrelationship between stresses discussed above to a very simple hand calculation. The magnitude of the maximum past lateral stress is given by:

$$\sigma_{max} = \sqrt{\frac{2P\gamma}{\pi}} \tag{2}$$

where

P = The line load applied by the compaction equipment

γ = The unit weight of the soil

An envelope analogous to that shown in Plate 6 is then constructed simply by following the *Kp* line to the maximum past lateral stress as given above and then dropping verticallv to the *Ko* line. Included in this construction is the assumption that 100 percent of the maximum past lateral stresses are retained by the soil.

Equation 2 assumes that passive failure occurs in the near surface soil. Thus, for small compaction equipment where this is not the case, this equation substantially overestimates the maximum past lateral pressure. It also models the load applied by the compaction equipment as a line load of infinite extent. However, for vibratory rollers with static weights of 10 tons or more, acting at a distance of 1 to 2 ft away from the wall, the results of the above equation agree very closely with those calculated as shown in Plate 6.

For the most conservative case, where 100 percent of the maximum previous value is retained, the example shown in Plate 6 would result in a critical depth of 9.8 ft, or by Ingold's equation, 11.7 ft. Thus, the compaction stresses would probably not be significant for typical gravity structures on the order of 40 to 50 ft high, but would be of considerable importance for structures less tnan 20 ft high.

5 Wall Friction

The effects of wall friction or shear in the backfill are specifically excluded from the present CE stability analyses. However, friction between the soil and the concrete wall does exist and it is common practice to allow for wall friction in the design of retaining walls (U.S. Army Corps of Engineers 1961). Wall friction is mobilized any time shear deformations occur along the contact between the wall and the soil. This deformation generally occurs from two sources. The placement of soil in lifts behind the wall causes shear along the wall as subsequent lifts cause compression of the underlying soil. Movement of the wall in response to lateral pressures also results in shear deformations taking place along the wall. The deformations required to mobilize wall friction are very small. Pressure cells installed at the Port Allen Lock (Kaufman and Sherman 1964) showed that enough movement occurred in filling and emptying the lock to mobilize measurable apparent wall friction.

One of the primary uncertainties in assessing the effects of wall friction on stability is the degree to which friction force may dissipate with time. Cohesive soils creep when subjected to a stress differential that is a significant portion of their shear strength, and some portion of the wall friction that develops may dissipate with time as a result of this creep. Granular soils are more likely to carry large stress differentials for long periods of time, but because of the vibrations and cyclic loadings typical of lock operations, the friction force in granular soils may also have some tendency to dissipate with time.

It is primarily as a result of this uncertainty that the current practice of excluding wall friction effects has evolved. However, it is common practice to design footing foundations on clay with a factor of safety of three relative to the shear strength of the soil with no concern for the foundation creeping with time. Footing foundations on sand are often designed with a factor of safety of two relative to the shear strength, again with no concern that the stress differentials will dissipate with time. Thus, it is reasonable to assume that the wall friction or shear friction in the backfill will be maintained with time provided that the resultant stress differential is not too large.

6 Conclusions

The present CE overturning analysis method is a simple method for checking the stability of a structure against overturning. In order to simplify the analysis, certain assumptions are made that do not necessarily reflect actual conditions. These assumptions introduce an unknown degree of conservatism into the analysis. This is most clearly illustrated when older structures, which have performed satisfactorily for years, are reevaluated and found to fall short of the present stability requirements. This report has discussed the simplifying assumptions and their effects on the analysis in three principal areas: base pressure distribution, strain compatibility, and wall friction.

Base Pressure Distribution

The present method of analysis contains numerous assumptions that result in a predicted base pressure which does not reflect the true state of stress in the foundation. The principal assumptions that affect the predicted pressure distribution are:

a. The structure is assumed to be flexible with respect to the foundation when it is, in fact, rigid. As a result, high edge stresses that occur under rigid structures are not considered.

b. The pressure distribution on the critical failure surface is assumed to be the same as the pressure distribution at the concrete-rock interface; in other words, the state of stress does not change with depth. In fact, stresses redistribute and dissipate quite rapidly with depth.

c. The weight of the backfill is assumed to have no effect on the state of stress on the critical failure plane. In fact, it provides a significant compressive stress in the immediate vicinity where tensile stresses will have a tendency to develop.

d. The rock mass is assumed to be incapable of carrying tensile stress. In some cases, the rock mass may have a small, but significant tensile strength.

e. Full hydrostatic uplift pressures are assumed to act on the portion of the base that is not in compression.

The above assumptions become important when a noncompressive zone appears in the predicted base pressure distribution. The first four assumptions lead to the prediction of a "noncompression" zone that is much larger than actually exists. The last assumption multiplies the effect of the overprediction by applying a high driving force to the overestimated length of the noncompressive zone. These assumptions restilt in an overly conservative assessment of the stability of a gravity structure.

Strain Compatibility

The lateral earth pressure applied to the wall is a function of the deformation conditions of the wall. For rock foundations with low to moderate elastic moduli, the elastic rotation of the structure may be sufficient to mobilize a significant portion of the shear strength of the backfill and the earth pressures applied to the wall may approach active earth pressures. Depending on the characteristics of the backfill material, this may significantly reduce the driving forces on the structure and increase the apparent stability considerably.

It is unlikely that a gravity structure on a rock foundation would ever deform sufficiently to mobilize the passive pressure on the front of the wall. Thus the present assumption of at rest pressure acting on the front of the wall is justified.

Where the elastic deformation of the foundation is limited by a high foundation modulus, at-rest earth pressures will be applied to the wall, and compaction-induced lateral stresses may be significant. For the type of compaction equipment likely to be used in the construction of gravity structures, the depth of influence of compaction stress may be on the order of 10 to 15 ft. Thus, for typical structures 40 to 50 ft high, the compaction-induced lateral stresses would not be particularly significant. However, for smaller structures, on the order of 20 ft high, compaction stresses would represent a significant portion ot the applied load.

Wall Friction

Wall friction effects are currently excluded from consideration in the CE analysis method because of a perceived tendency for the friction force

to dissipate with time. However, every foundation that applies a load to soil creates a moderate stress differential which is carried indefinitely. Thus, it is reasonable to expect that friction forces would be maintained indefinitely provided that t..ey represent a sufficiently small fraction of the shear strength of the backfill material.

7 Recommendations

Recommendations are presented in two general areas: (a) suggested modifications to the CE method of analysis that can be considered on the basis of this study; and (b) areas where further study may result in additional improvements to the CE method of analysis.

Suggested Modifications

Because the base pressure distribution is the primary criterion by which overturning stability is assessed, it is recommended that the assumptions made in calculating the base pressure be modified to more closely fit actual conditions. At the very least, the distribution of stress with depth and the effects of the backfill on both sides of the wall should be included in the assessment of the base pressure. Simple elastic solutions are available for calculating pressure distribution with depth for a wide variety of uniform loading conditions. While they do not model the high edge stresses that occur under rigid structures, they still represent a significant improvement over the present methods.

The assumption with regard to the lateral earth pressure applied to the structure should be checked against the known deformation conditions. It is relatively easy to calculate the elastic rotation of the structure and determine if a reduction in the earth pressure is warranted on the basis of the deformation conditions.

Wall friction effects should be included in the analysis at least for short-term loading conditions such as flood loads and maintenance conditions and should be considered for long-term loading conditions as well. If the wall friction is limited to a fraction of its fully mobilized value, it can be maintained indefinitely. Current conventions with regard to footing foundation design indicate that an allowable friction force of one-third the fully mobilized value is a reasonable guideline.

Future Studies

The assumption that full uplift pressure is applied to the area of the base not in compression is not always confirmed by field measurements. It is recommended that all available data on piezometric pressure measured at the heel of gravity structures be collected and evaluated with respect to the loading conditions on the structure. On the basis of this evaluation, a more realistic convention could be established for the consideration of uplift pressures.

Future studies should also consider some of the same questions discussed in this report with regard to sliding stability. These would include the effects of the actual pressure distribution on sliding stability, how to include strain compatibility considerations in the limit equilibrium method, and the potential for updating and modifying the shear friction method as an alternative to the limit equilibrium method.

For gravity structures on rock foundations, considerations of the site geologic conditions are an important factor in assessing the stability of the structure. Future studies should also address the geologic characterization of a site, including the identification of critical planes of weakness, adverse geometries, transmission of uplift pressures, and likely modes of failure. These issues will affect not only the shear strength parameters used for design, but also the validity of the fundamental assumptions made in the stability analysis.

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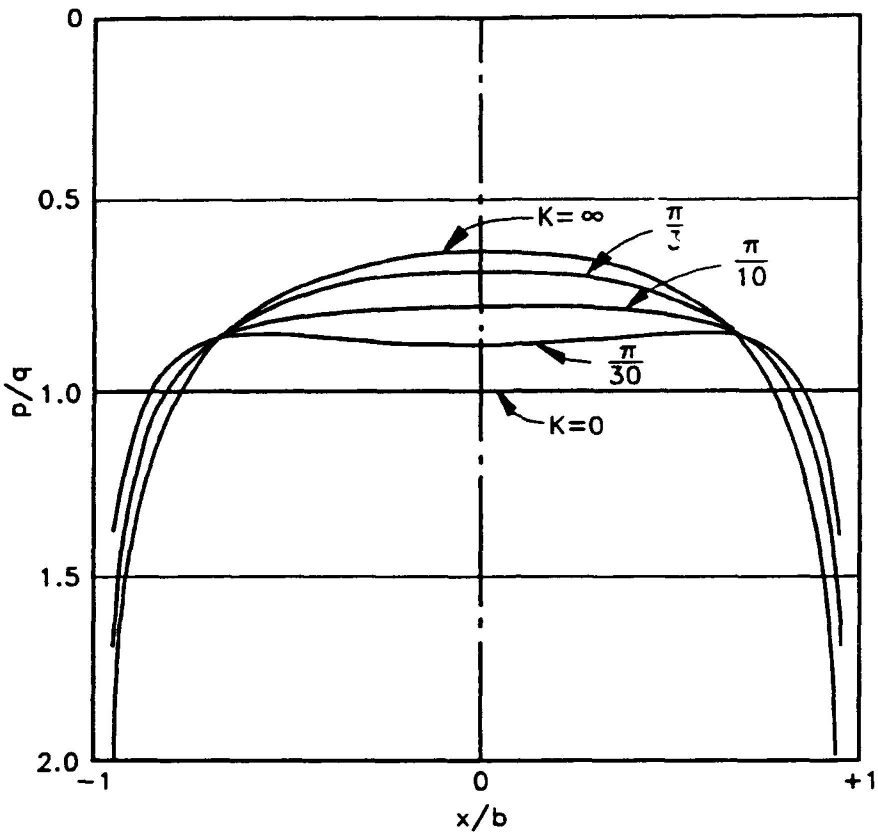


p = CONTACT STRESS

q = APPLIED LOAD

x = DISTANCE FROM CENTER OF STRUCTURE

b = HALFWIDTH OF BASE OF STRUCTURE

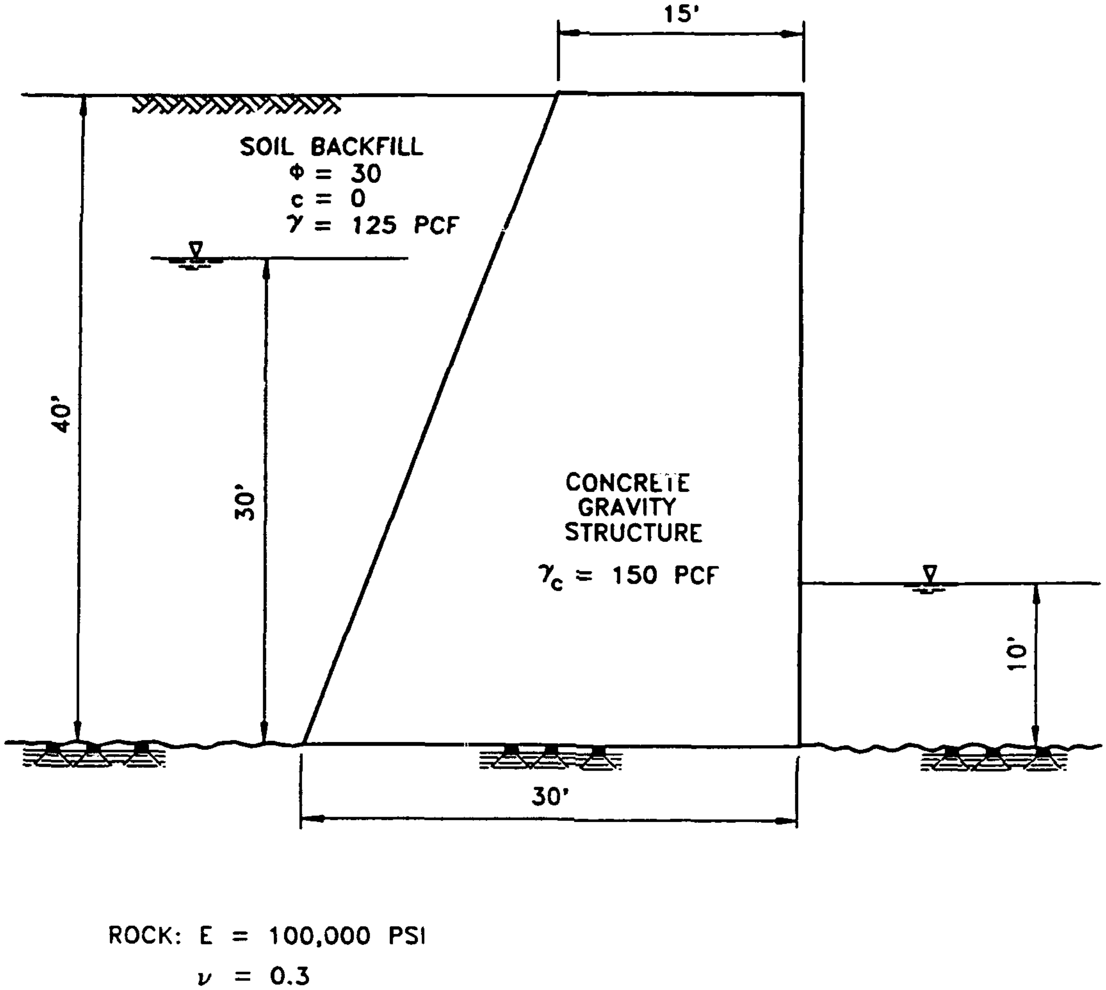
K = RELATIVE STIFFNESS

$$= \frac{1}{6} \cdot \frac{1-\nu_f^{2}}{1-\nu_s^{2}} \cdot \frac{E_s}{E_f} \left(\frac{t}{b}\right)^3$$

STABILITY STUDY

WATERWAYS EXPERIMENT STATION

CONTACT STRESS DISTRIBUTION VS. RELATIVE STIFFNESS



STABILITY STUDY
WATERWAYS EXPERIMENT STATION

TYPICAL GRAVITY
STRUCTURE CONFIGURATION

Plate 2

CONTACT STRESS UNDER RIGID BODY

σ_c = VERTICAL LOAD COMPONENT + MOMENT COMPONENT

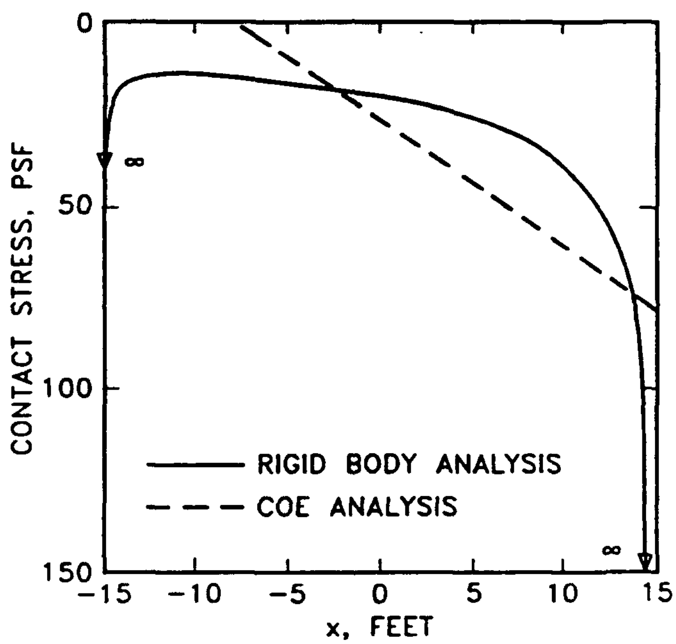
$$\sigma_c = \frac{P_o}{\pi\sqrt{b^2-x^2}} + \frac{2Mx}{\pi b^2 \sqrt{b^2-x^2}}$$

WHERE: P = VERTICAL LOAD (UNITS OF FORCE PER UNIT LENGTH OF STRUCTURE, e.g., LBS/FT)

M = MOMENT (UNITS OF MOMENT PER UNIT LENGTH OF STRUCTURE, e.g., FT-LBS/FT)

b = HALFWIDTH OF BASE OF STRUCTURE

x = DISTANCE FROM CENTER OF STRUCTURE

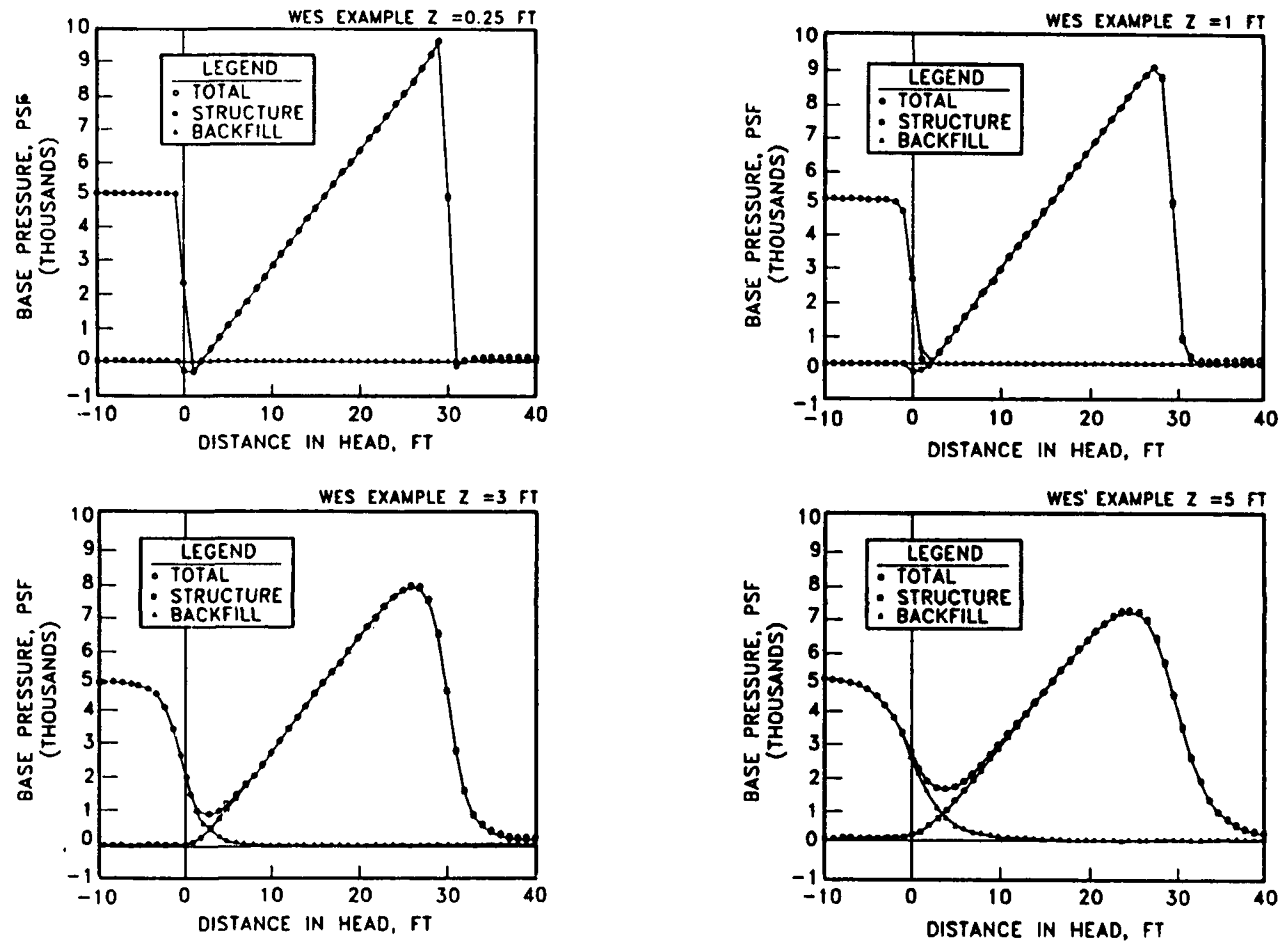


NOTE: SEE FIGURE 2 FOR TYPICAL CONFIGURATION.

STABILITY STUDY

WATERWAYS EXPERIMENT STATION

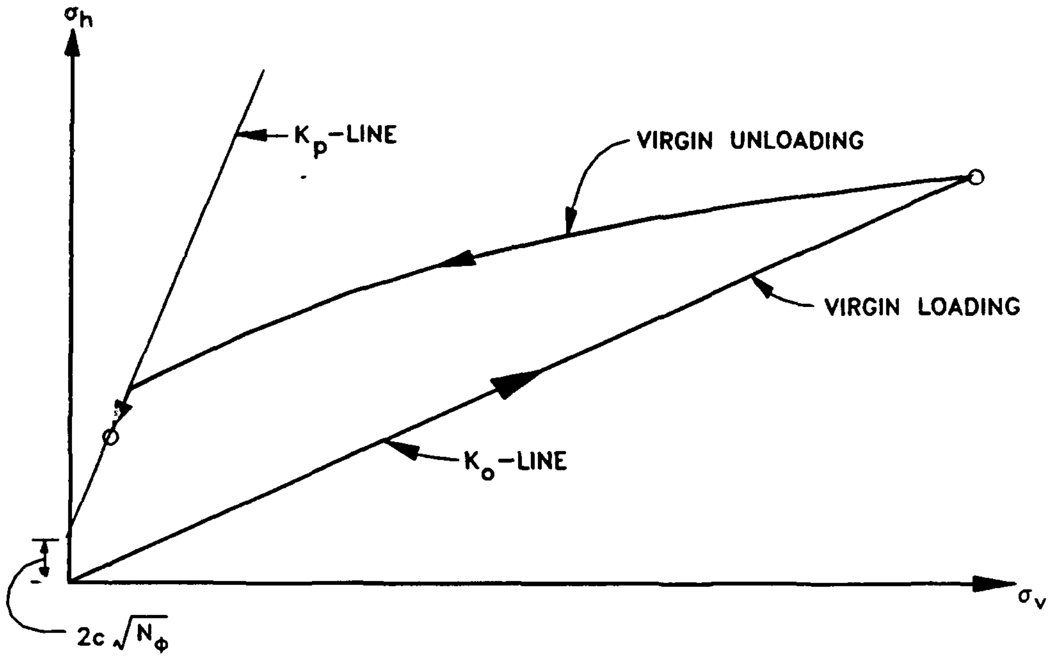
CONTACT STRESS DISTRIBUTION FOR TYPICAL CONFIGURATION



STABILITY STUDY

WATERWAYS EXPERIMENT STATION

BASE PRESSURE DISTRIBUTION WITH DEPTH

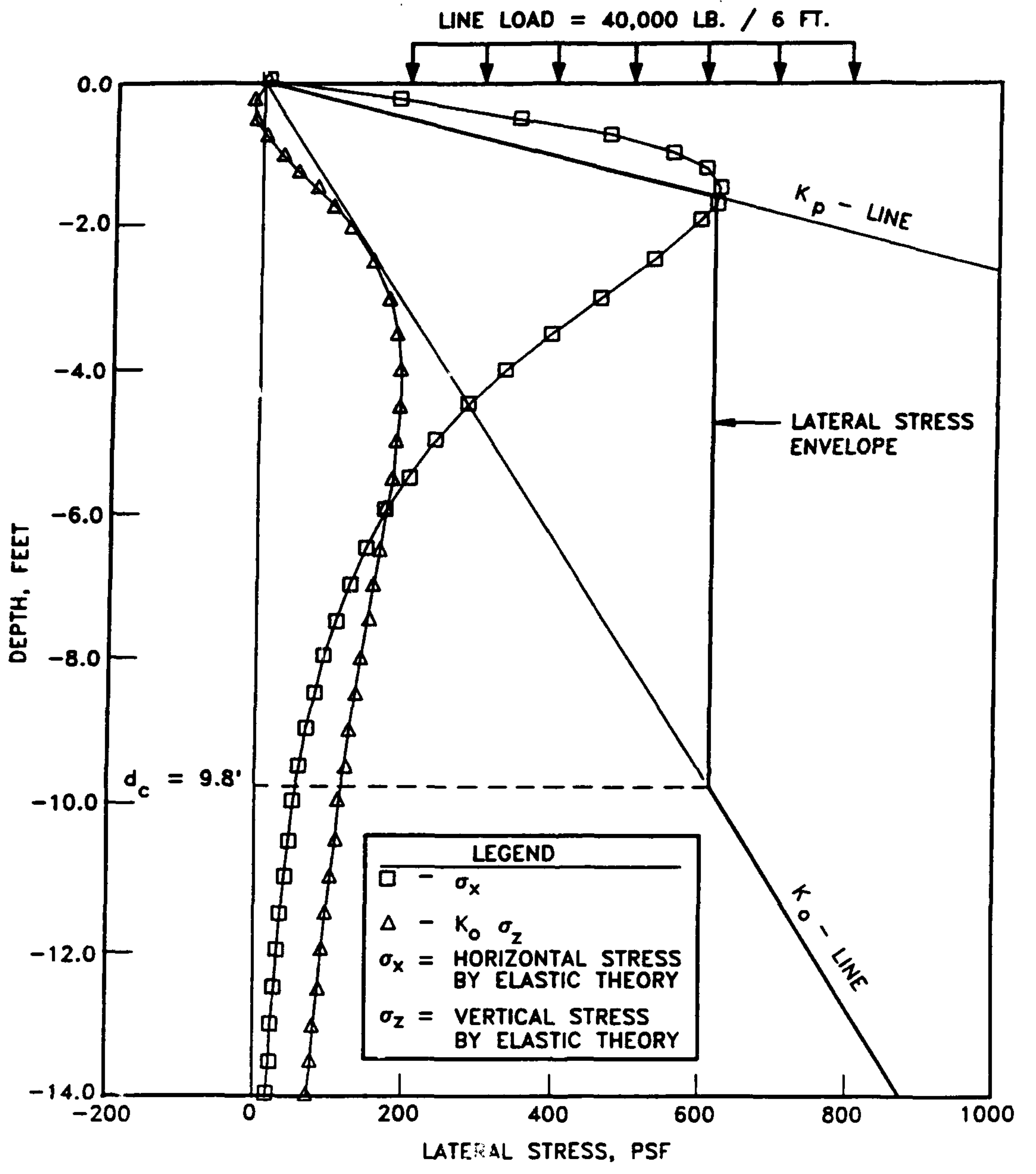


STABILITY STUDY

WATERWAYS EXPERIMENT STATION

STRESS PATH

LOADING/UNLOADING MODEL



STABILITY STUDY
WATERWAYS EXPERIMENT STATION

COMPACTION INDUCED
LATERAL STRESSES

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13. ABSTRACT (*Maximum 200 words*)

This report includes a discussion of the present method of overturning analysis and a discussion of the underlying assumptions; the contact stresses under the base of the structures; and the effects of strain compatibility, compaction-induced lateral stresses, and wall friction on the driving and resisting moments. Recommendations are made for modifications to the present method of analysis, and areas for future studies where additional improvements could be made are discussed.

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